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EARTHQUAKE RESEARCH LABORATORY

Response of a Structure to an Explosive-Generated Ground Shock

by

D. E. Hudson, J. L. Alford, and

G. W. Housner

A REPORT ON RESEARCH CONDUCTED UNDER

CONTRACT WITH THE OFFICE OF NAVAL RESEARCH

September 1952

RESPONSE OF A STRUCTURE

TO AN EXPLOSIVE - GENERATED GROUND SHOCK

by

D. E. HUDSON, J. L. ALFORD and
G. W. HOUSNER

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RESPONSE OF A STRUCTURE

TO AN EXPLOSIVE-GENERATED GROUND SHOCK

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ABSTRACT

Measurements were made of ground accelerations and the resulting building accelerations at a point very near a large quarry blast. It is shown that, in the case of simple buildings, the building acceleration may be calculated with satisfactory accuracy from a knowledge of the ground acceleration.

The response of the test building to the ground acceleration of a typical strong-motion earthquake was computed, and it was found that the resulting accelerations were in excess of those usually provided for in earthquake-resistant design. It is concluded that the satisfactory performance of well-designed structures during strong earthquakes may have two explanations: first, that vibration energy is dissipated by stresses in excess of the elastic limit, with the result that hidden damage may occur; and second, that ordinary buildings may have sources of strength which are not taken into account in their design.

MEASURED RESPONSE OF A STRUCTURE TO AN EXPLOSIVE GENERATED GROUND SHOCK

I. Introduction

The study of the response of structures to strong earth shocks is hampered by the difficulty of carrying out experimental investigations. Since there is no control over the occurrence of earthquakes it is difficult to plan a satisfactory experimental program for measuring the behavior of structures during strong ground motion. The only available procedure is to install measuring instruments in buildings and then wait for a strong earthquake to occur in that vicinity. Instruments have been installed in certain buildings in California but as yet no strong motion has been recorded by them. Earthquakes, of course, can be simulated by the detonation of explosives but for the purpose of studying the response of buildings it is necessary to have strong ground motion which requires a large quantity of explosive. Although from time to time large quantities of explosives are detonated, usually for military purposes, it is very seldom that a building on which it is suitable to make measurements is located sufficiently close to the explosion to be strongly shaken.

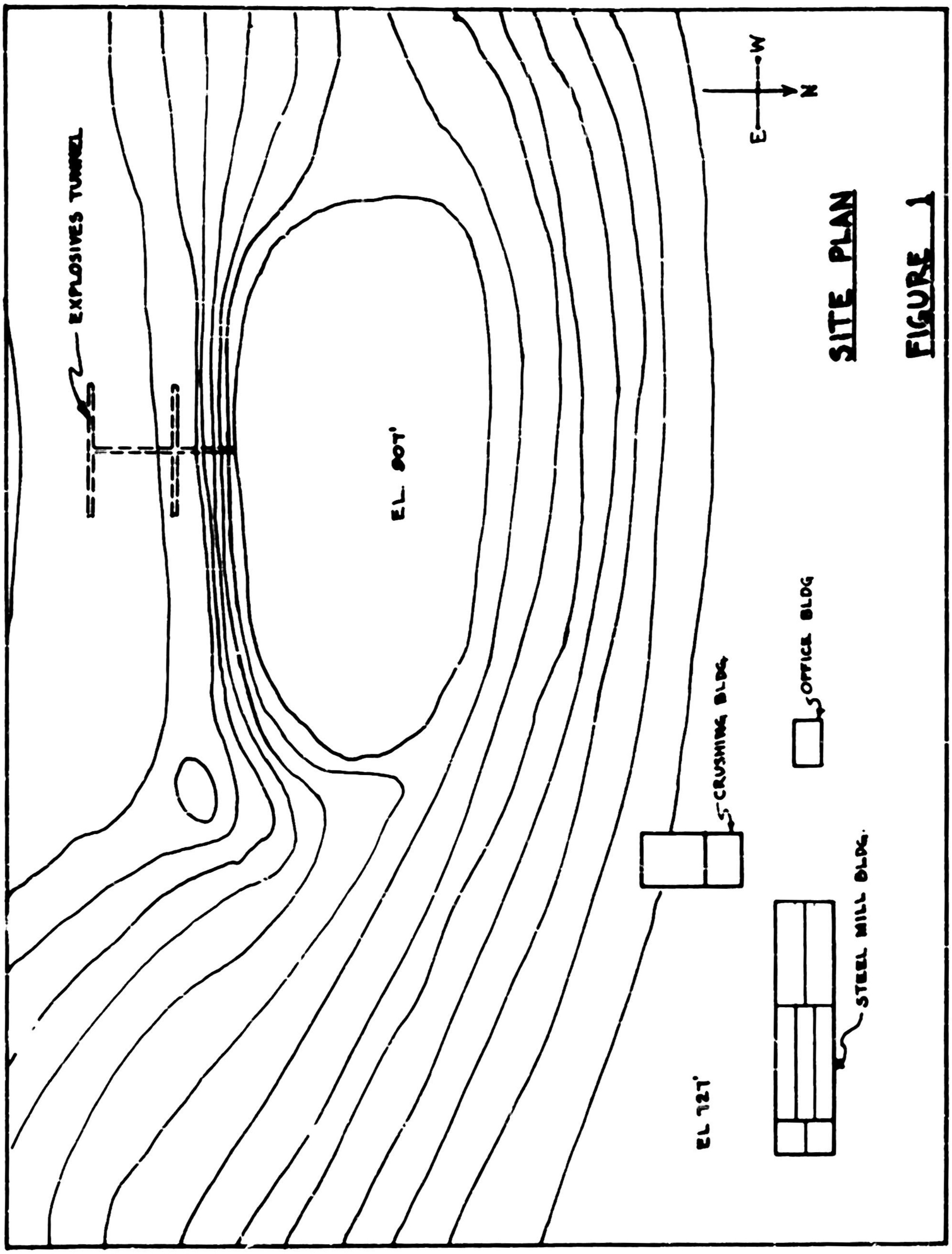
In view of the foregoing difficulties it was fortunate that the Minnesota Mining and Manufacturing Company planned to detonate a large quantity of explosives on July 26, 1952 at their rock quarry near Corona, California and that there was a well designed and constructed building sufficiently near the point of detonation so that it would be subjected to reasonably strong ground motion. The opportunity to make measurements during this explosion thus made it possible to measure the ground shock and so determine the character and intensity of motion generated by the detonation of a large quantity of buried explosive, and also to measure the motion of a building and compare this response with the ground motion that produced it. Since the explosive-generated shock is just a synthetic earthquake these measurements throw light on the behavior of structures during natural strong-motion earthquakes.

II. Layout of the Site

The quarry operated by the Minnesota Mining and Manufacturing Company, at which the explosive was detonated, is located approximatel five miles south of the town of Corona, California. The site of the quarry is in a canyon between low hills from one of which the rock is quarried. Figure 1 shows the topography of the quarry site and the location of the mill building in which measurements were made. The building is at the eastern end of the quarry grounds and the measuring instruments that were used were located at the eastern end of the building. The structure stands on the flat bottom of the canyon and the explosive charge was detonated at a horizontal distance of 370 yards from the east end of the building and 60 yards above the elevation of the ground upon which the building stands. Figure 2 shows the relation between the end of the building where the measuring instruments were located and the point on the hillside where the explosives were detonated. It is seen from this figure that because of the difference in elevation the shortest distance through the ground from the point of the explosion to the point where the measurements were made was not a straight line.

The ground in the vicinity of the quarry is a rather solid, moderately fissured, friable rock (dacite porphyry). The purpose of the blast was to loosen up a portion of the hill and to break it into relatively small pieces of rock. These are ground to a fine size, processed and eventually end up as the outer coating of roofing materials.

In preparation for the blast a horizontal shaft approximately five feet in diameter was driven into the side of the hill approximately 170 feet with lateral galleries extending from it. The explosive, 370,000 pounds of Nitramon plus 1500 pounds of Primer, was placed in the shafts and back-filled to give a tamped explosion. The explosion worked as planned, slightly lifting and moving out a portion of the hill with no scattering of rocks.



SITE PLAN

FIGURE 1

SECTION THROUGH SITE

FIGURE 2

III. Description of the Building

The building in which the measurements were made is a steel-frame mill building constructed in 1947. It has corrugated iron siding and roofing and its plan dimensions are 65 feet by 285 feet. A photograph of the building is shown in Figure 3, looking west. The face of the hill where the explosives were detonated is visible just to the left of the building. The east end of the building is taller than the west end which is a typical one-story mill building. Details of the framing of the one-story portion are shown in Figure 4. The east end of the building has plan dimensions of 65 feet by 37 feet. The overall height of this portion of the building is 103 feet above the ground floor. At an elevation of 45' - 8" above the ground floor there is a 6" thick concrete floor slab (65' x 37'). This floor slab is at approximately the same elevation as the roof of the one-story portion so that the heaviest mass of the building is concentrated at this elevation. Above this floor slab there is the steel wall and roof framing, the corrugated iron siding and roofing, and a steel deck floor at an elevation 84' - 6" above the ground floor. The framing of the east wall is shown in Figure 5. The ground floor of the building is an 8" reinforced concrete slab. There is a single basement beneath the one-story portion of the building and a double basement beneath the east end of the building, with 18" thick reinforced concrete walls. The sub-surface structure of the building thus is essentially a concrete box upon which the steel frame building is standing. A plan of the building is shown in Figure 6.

The heavy concrete floor at elevation 45' - 8" in the east end of the building has a weight of 180,000 pounds. It is thus by far the most effective element of the building in producing stresses when the building is subjected to ground motion. The building is essentially an oscillator with a heavy mass at elevation 45' - 8", modified somewhat by the effect of the adjacent portion of the building.

Figure 3. Looking west at steel-frame mill building. Explosion occurred at face of hill just left of upper part of building.

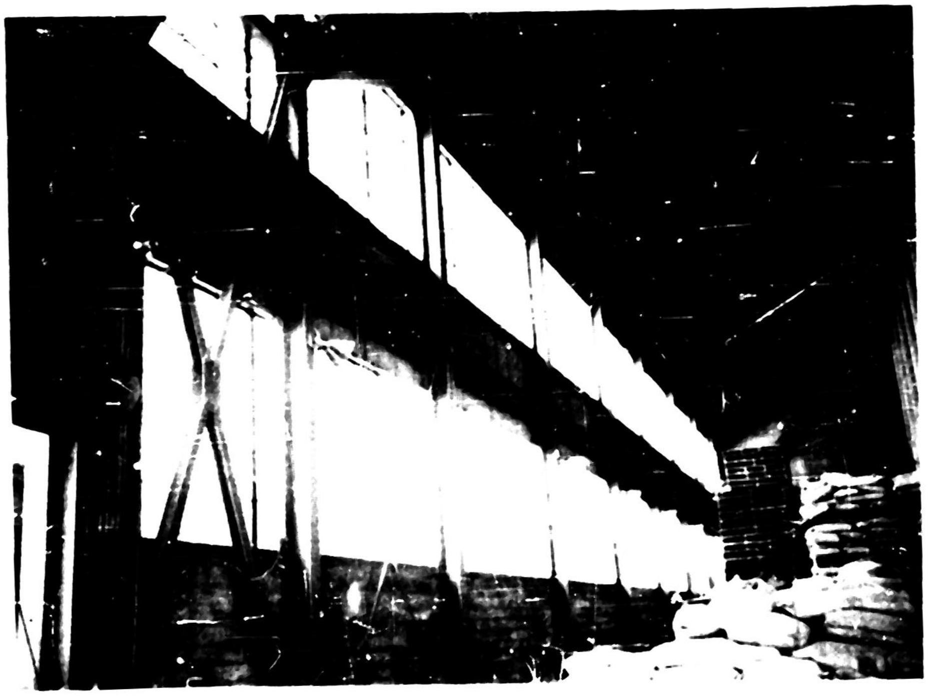


Figure 4. South wall of mill building.

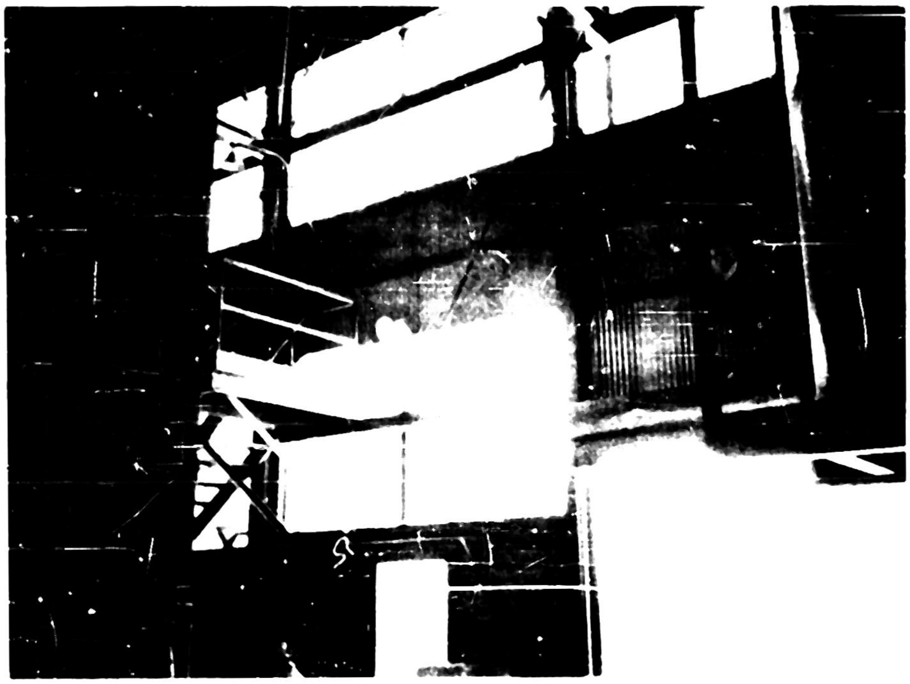
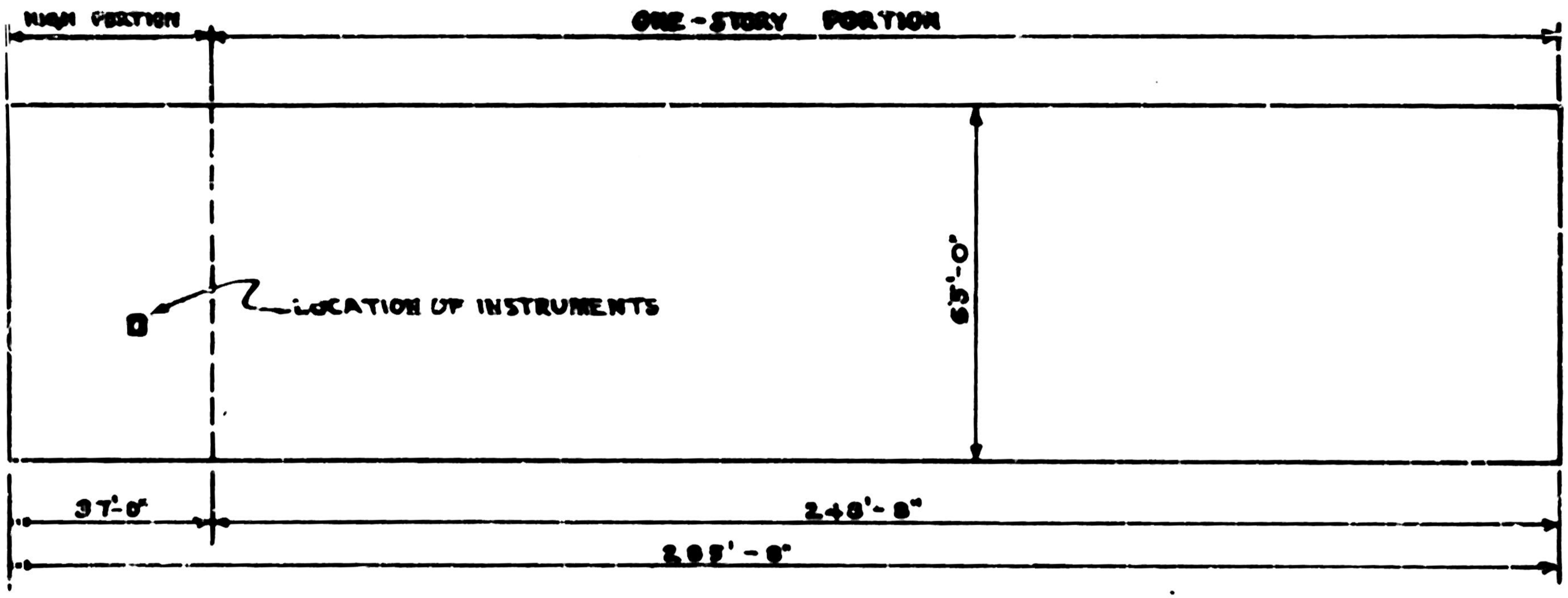


Figure 5a. East wall of mill building.

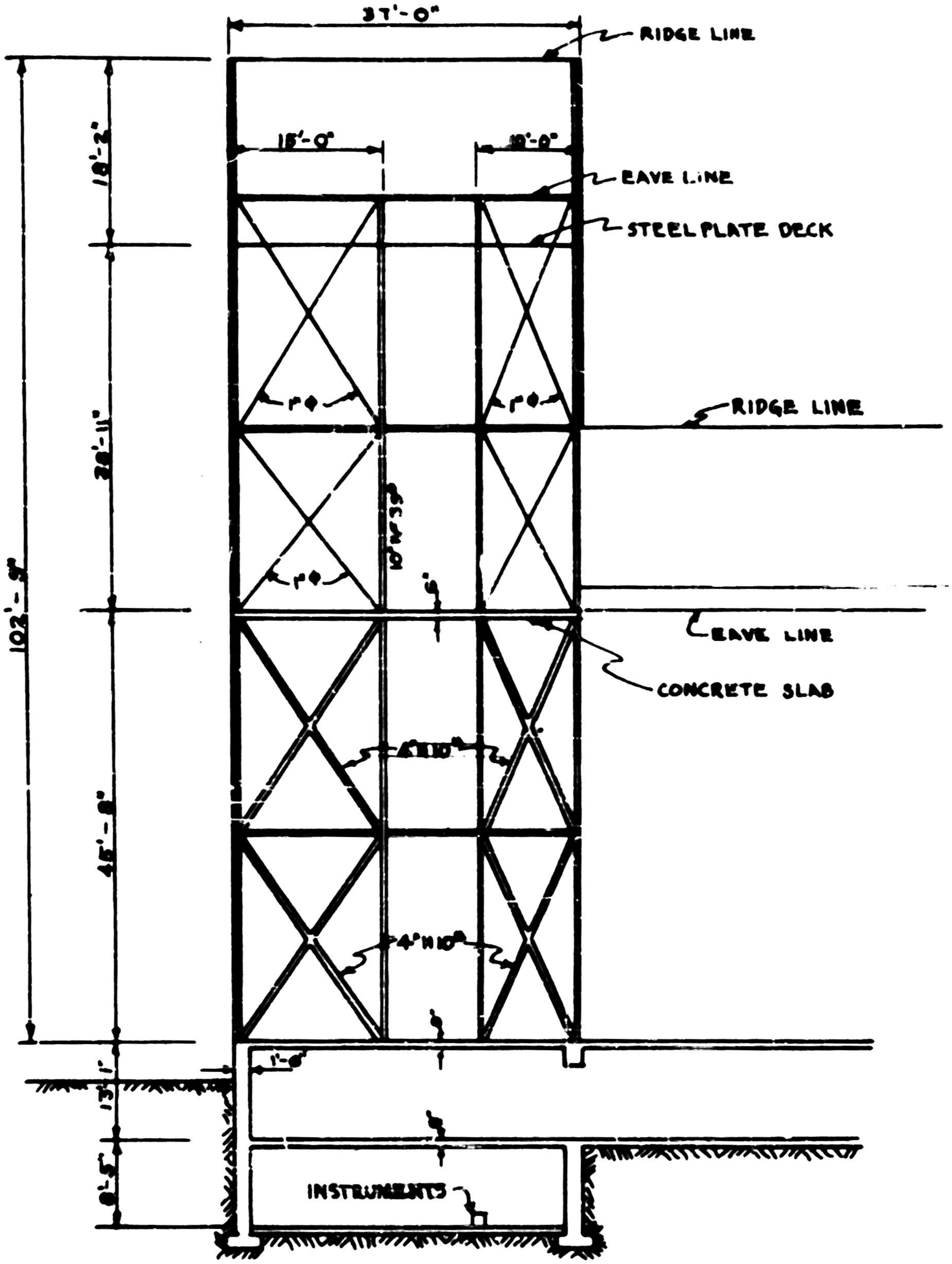
Figure 5b. South wall of mill building.



PLAN OF BUILDING

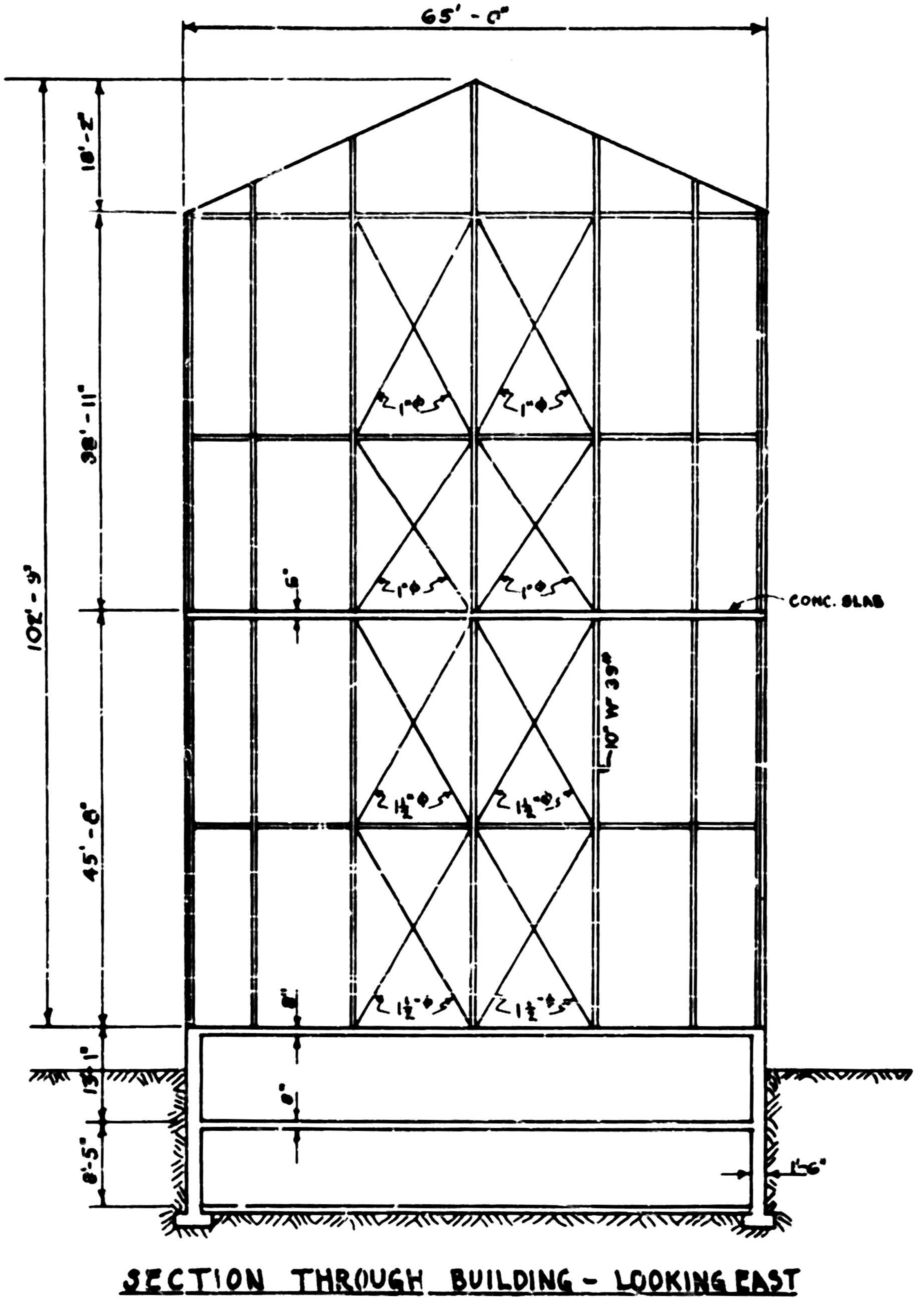
FIGURE 6

An east-west section through this portion of the building is shown in Figure 7. There are two sets of X-bracing in both the north and south walls which provide lateral bracing for the elevated concrete floor slab. This bracing is made of 4" - H - 10# sections. The steel columns in this portion of the building are 10" WF 39 # sections. The east wall of the building is framed similarly except that it has a double set of X-bracing made of 1 1/2" round rods as shown in Figure 8.



SECTION THROUGH BUILDING - LOOKING SOUTH

FIGURE 7



SECTION THROUGH BUILDING - LOOKING EAST

FIGURE 8

IV. The Instruments.

Two different types of instruments were used to measure the motions produced by the shock. One type was an electrical instrument with direct-inking recording and the other type was a mechanical instrument with optical recording. The instruments were set up in the sub-basement of the east end of the building and on the concrete floor slab at elevation 45' - 8" above the ground floor level.

On the upper floor the instruments were located 25 feet from the north wall and 24 feet from the east wall. The accelerometer pick-up was bolted to the concrete slab by means of a small angle and the accelerometer was oriented so as to pick up motion in an east-west direction. The accelerometer was a William Miller Corporation Type 402-C, Serial No. 5. This instrument is a small seismic type accelerometer with a natural frequency of approximately 80 cycles per second. It has fluid damping (60% of critical) and a variable reluctance balanced-armature type measuring element. The overall size of the accelerometer is 1 x 1 1/2 x 2 1/2 in.

The signal from the accelerometer was fed into a carrier type amplifier system consisting of a Brush Development Company Type BL-320 "Universal Analyzer", Serial No. 415. This instrument includes an input bridge circuit, a 2000 cyc/sec. oscillator, an A. C. Amplifier, demodulator-discriminator circuit, and D. C. Power Amplifier.

The output of the oscillator-amplifier was fed into a Brush Development Company, Type BL-202, Direct Inking Oscillograph, Serial No. 3027. The Amplifier-Oscillograph combination has a flat frequency response from 0 to 100 cycles per second. A photograph of this instrumental set-up is shown in Figure 9.

On the floor of the sub-basement, directly beneath the above described instruments, there was a duplicate set-up using a William Miller Corporation Type 402-C accelerometer, Serial No. 139; a Brush Development Company Type BL-320 "Universal Analyzer", Serial No. 429, and a Brush Development Company Type BL-202

Figure 9a. Location of instruments on upper floor.

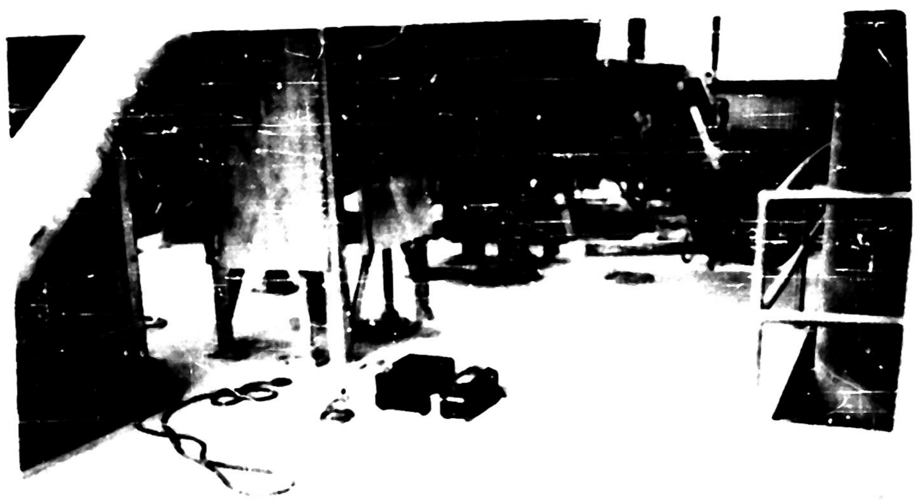


Figure 9b. Location of instruments on upper floor.

Direct-Inking Oscillograph Serial No. 1189. The accelerometer was bolted to the concrete floor slab and was oriented in an east-west direction. A photograph of the installed instruments is shown in Figure 10.

During the blast the electrical power was turned off at the quarry so the instruments were operated on a power supply furnished by a gas-driven 110 volt-60 cycle generator.

The accelerometers were calibrated immediately before and after the test by removing them from their bracket mountings and rotating them through 90° thus applying a 1 g acceleration. The calibrated sensitivity of the upper accelerometer and recorder was 0.0356 g equals 5 mm. (five small divisions) on the recording paper. The basement accelerometer and recorder had a calibrated sensitivity of 0.100 g equals 5 mm.

The paper speed of the upper recorder was one second equals five millimeters and the speed of the basement recorder was 0.20 seconds equals five millimeters.

To record the shock the instruments were put into operation one minute before detonation with continuous recording until one half minute after.

In addition to the above described instruments a three-component accelerometer belonging to the U.S. Coast and Geodetic Survey was installed in the sub-basement adjacent to the other instrument. The case containing this instrument was bolted to the floor with its longitudinal axis oriented east-west. This instrument was one of the standard seismometers used by the U.S.C. G.S. for recording strong earthquakes. It was the instrument which is normally in operation on the campus of the California Institute of Technology.

The unit contains three accelerometers of the torsion-suspension type which record two horizontal and one vertical component of motion. The response of the accelerometers is recorded optically on photographic paper with automatic time recording. The natural frequency of the accelerometers is 12 cycles per second and they have magnetic damping (approximately 60% of critical). A photograph of the instrument installed in the sub-basement, with its cover lifted, is shown in Figure 11.

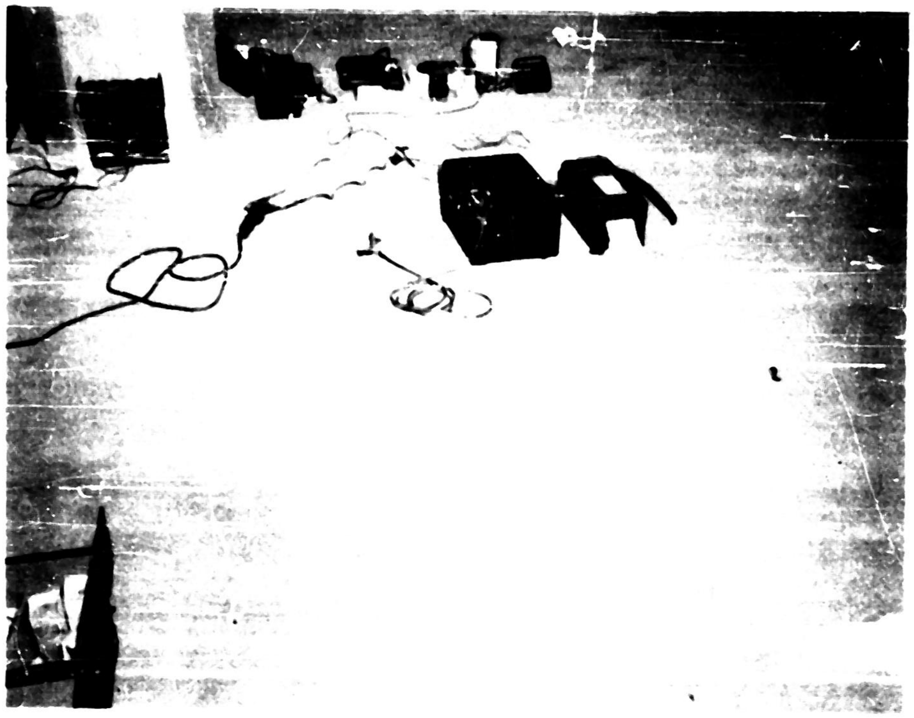


Figure 10. Location of instruments on floor of sub-basement.

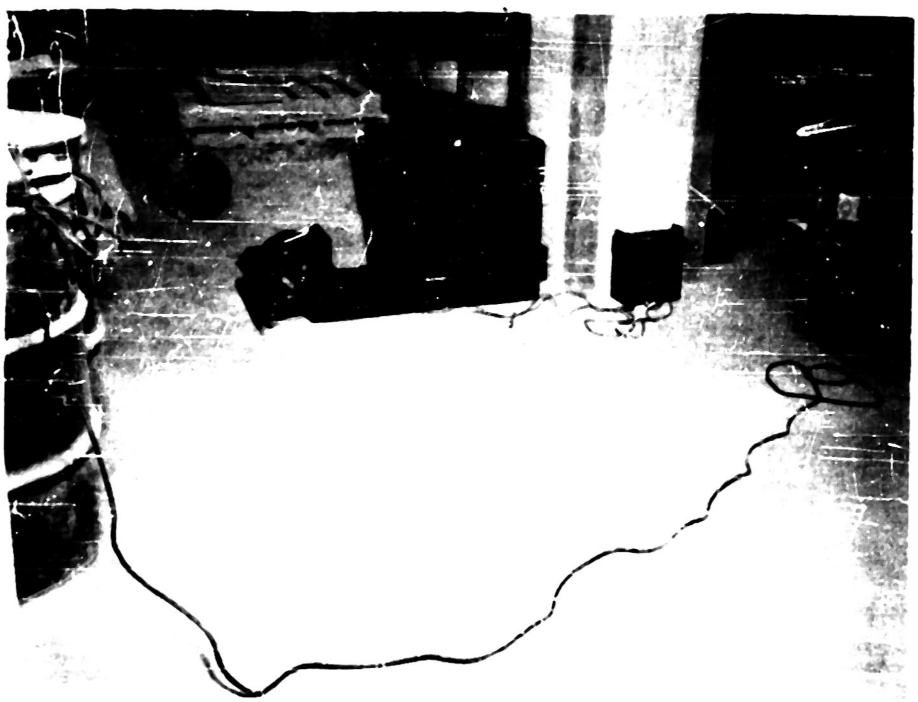


Figure 11. Location of U.S. Coast and Geodetic Survey accelerometer on floor of sub-basement.

V. Recorded Ground Motion

Photographs of the records of the ground motion, that is, the motion of the floor of the sub-basement, are shown in Figures 12 and 13. The same records drawn to a common scale are shown in Figures 14, 15, 16 and 17. Comparing Figures 14 and 15, which represent the same ground motion recorded by two different instruments, it is seen that there is very close agreement between the two records. Practically the same maximum accelerations were recorded and the shapes of the two records are virtually the same. The fact that the two instruments are so dissimilar, one being an electrical instrument with mechanical pen recording and the other a mechanical instrument with optical recording, and that they both gave the same record for the ground motion is a good check on the reliability of the instruments.

From Figures 14, 15, 16 and 17, it is seen that the maximum recorded ground acceleration in the east-west direction was 8% g, the maximum acceleration in the north-south direction was 10.5% g and in the vertical direction the maximum acceleration was 12.4% g. Accelerations of this magnitude correspond to the ground motion experienced during a moderately strong earthquake. For purposes of comparison there is shown in Figure 18 a portion of a horizontal component of motion recorded at Hollister, California during the earthquake of March 9, 1949. Comparing the records, it is seen that the ground motion recorded during the explosion is very similar to the recorded earthquake motion. The ground motion produced by the detonation of the explosives is thus practically the same as a moderately strong but a very short duration earthquake.

The recorded vertical ground motion during the blast shows distinctly sharper peaks, that is, higher frequency components, than the horizontal ground motion. This also agrees with the strong-motion earthquake records, which invariably show the vertical motion to have significantly higher frequency components than the horizontal motion.

The instruments that recorded the motion were oriented

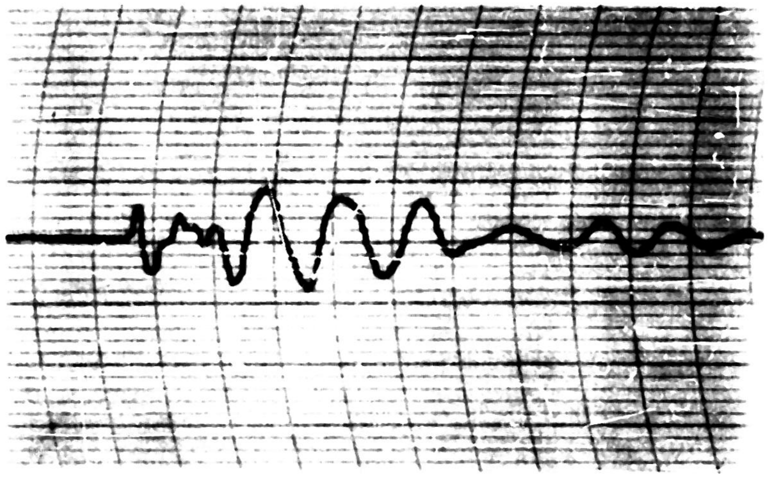


Figure 12. Photograph of east-west acceleration record from Brush recorder on floor of sub-basement.

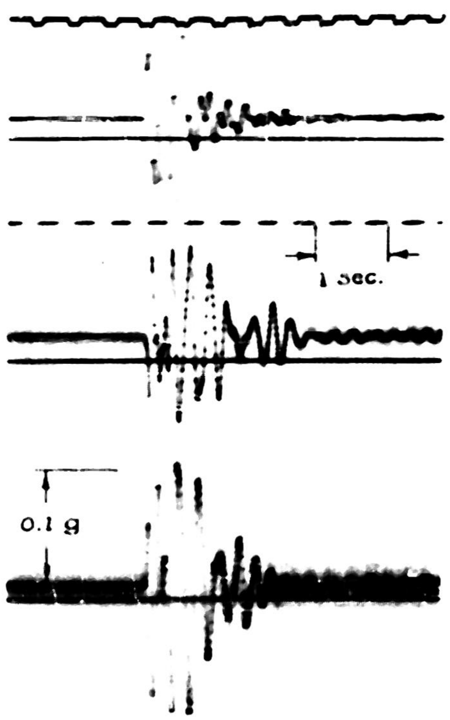


Figure 13. Photograph of records from U.S.C.G.S. accelerometer on floor of sub-basement.

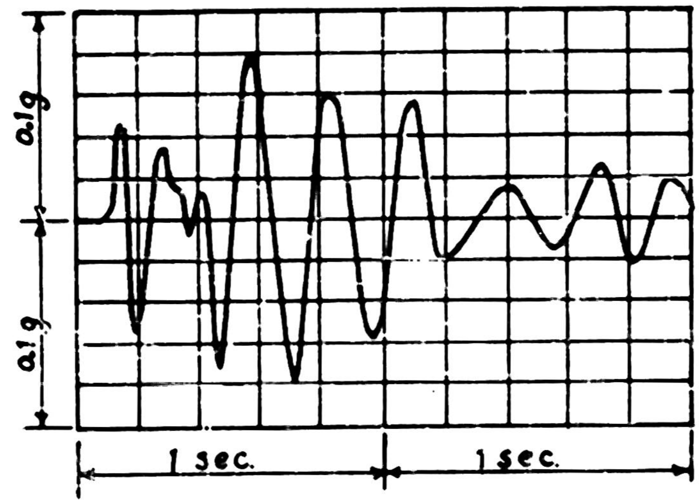


Figure 14. East-west ground acceleration obtained with Brush recorder.

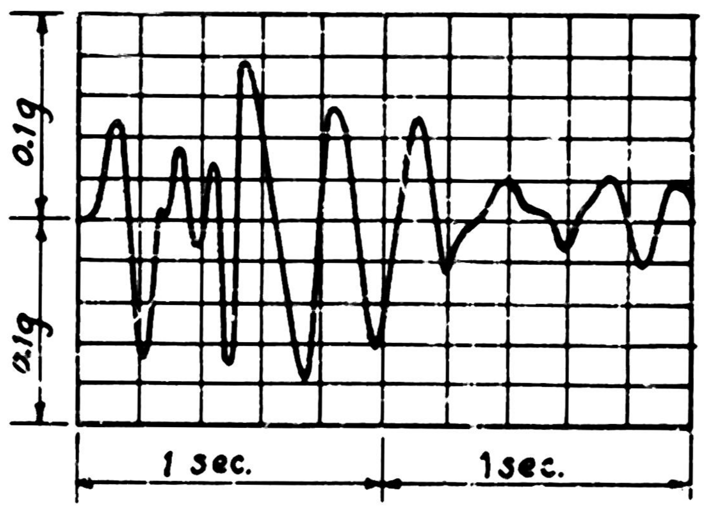


Figure 15. East-west ground acceleration obtained with U.S.C.G.S. accelerometer.

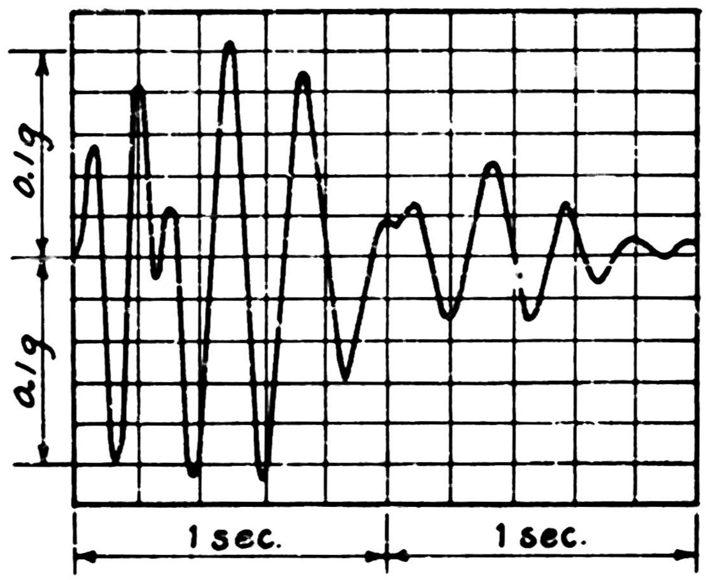


Figure 16. North-south ground acceleration obtained with U.S.C.G.S. accelerometer.

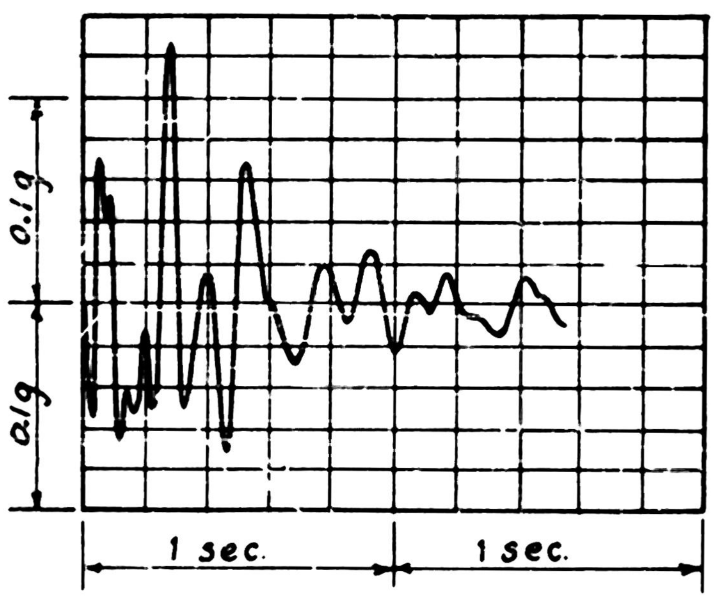


Figure 17. Vertical ground acceleration obtained with U.S.C.G.S. accelerometer

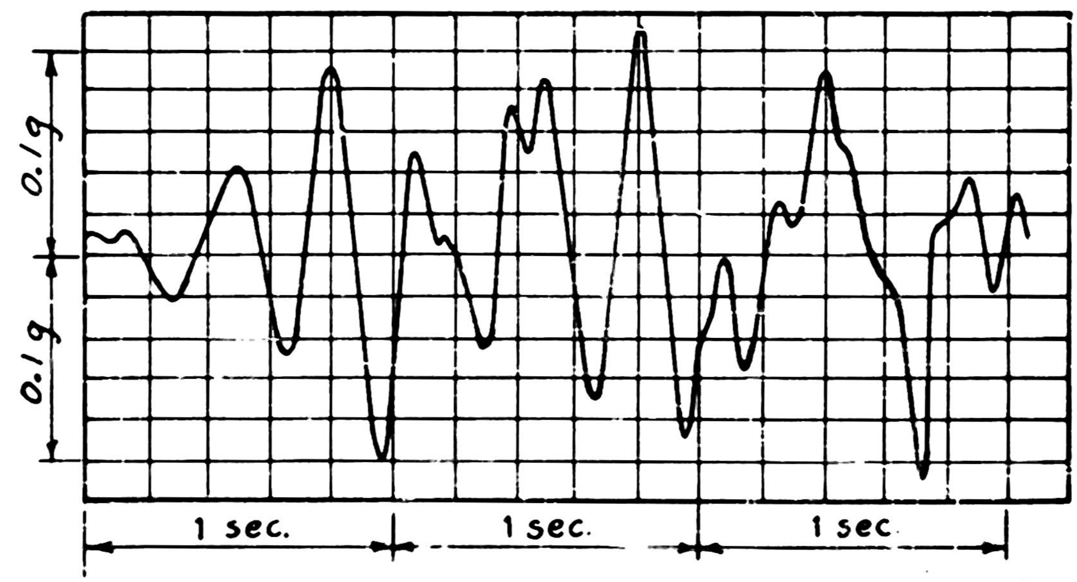


Figure 18. Initial portion of the north-south ground acceleration recorded at Hollister, California during the earthquake of March 9, 1949.

parallel to the axes of the building, which were approximately 45° from the direction of the blast. The most intense ground motion is in a radial direction from the point of detonation and when the two recorded components are resolved to give the radial and tangential components of acceleration the accelerograms shown in Figures 20 and 21 are obtained. It is seen that practically all of the motion was in the radial direction and only relatively small accelerations in the tangential direction. The maximum resolved acceleration (Figure 20) is 13% g.

For purposes of comparison the most intense ground motion recorded in California is shown in Figure 19. This is a horizontal component of the ground motion recorded at El Centro, California during the shock of May 18, 1940. The maximum acceleration recorded here was 33% g as compared to 13% g recorded during the blast. The duration of the intense ground motion during the El Centro shock was 25 seconds as compared to the one second duration of the strong ground motion produced by the blast.

The effect of a ground shock upon structures is conveniently summarized in the spectrum, which has been described in detail in an earlier report.* Briefly, the acceleration spectrum consists of the maximum acceleration experienced by a single degree of freedom oscillator when subjected to the given ground motion plotted as a function of the natural period of the oscillator. Such a spectrum was computed for the quarry blast, by means of the electric analog computor, and is shown in Figure 22. Spectra are shown for structures having 2%, 5% and 10% of critical damping. Several points about these spectra are noteworthy: first, the significant response is confined to a narrow range of periods at the short-period end of the spectrum; second, the maximum response (with 2% of critical damping) is approximately 0.5 g.

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*) J. L. Alford, G. W. Housner, and R.R. Martel, "Spectrum Analyses of Strong-Motion Earthquakes", First Tech. Report, ONR Project NR-081-095, Pasadena, California, August 1951.

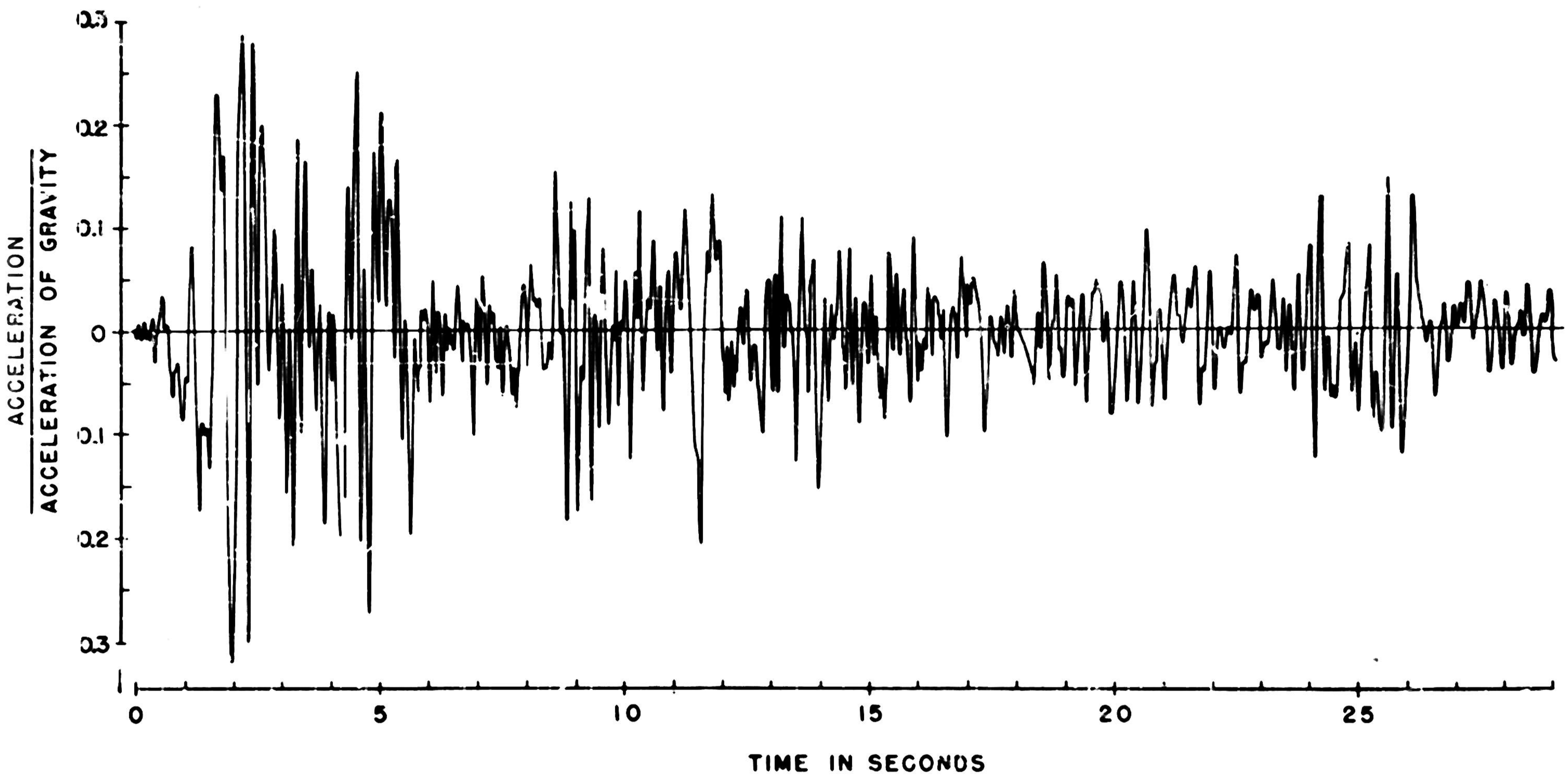

Figure 19. Accelerogram for El Centro, California; earthquake of May 18, 1940. Component N-S.

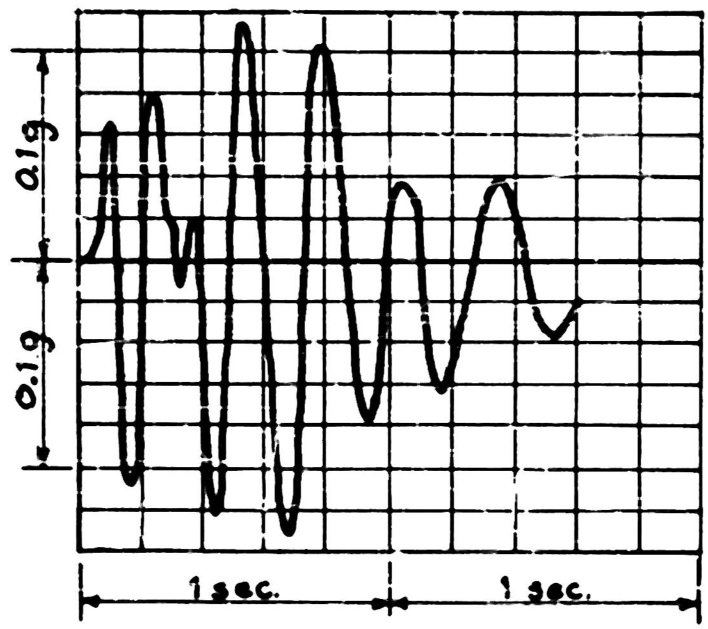

Figure 20. Ground acceleration resolved along a radial line from the blast source.

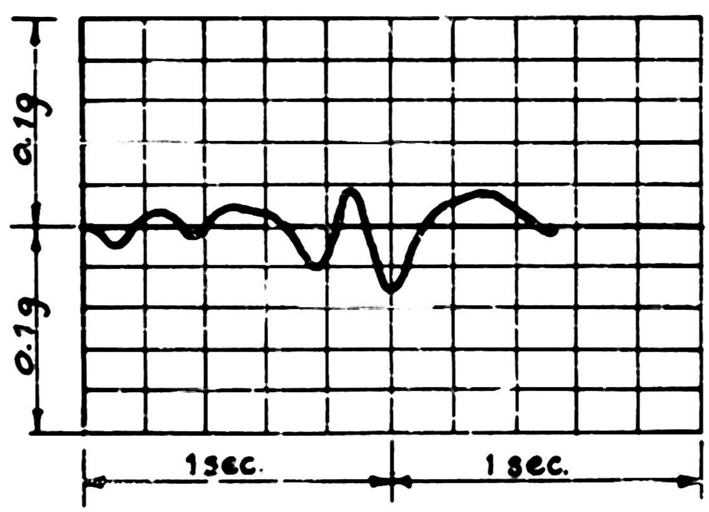

Figure 21. Ground acceleration resolved along a line perpendicular to the radial direction.

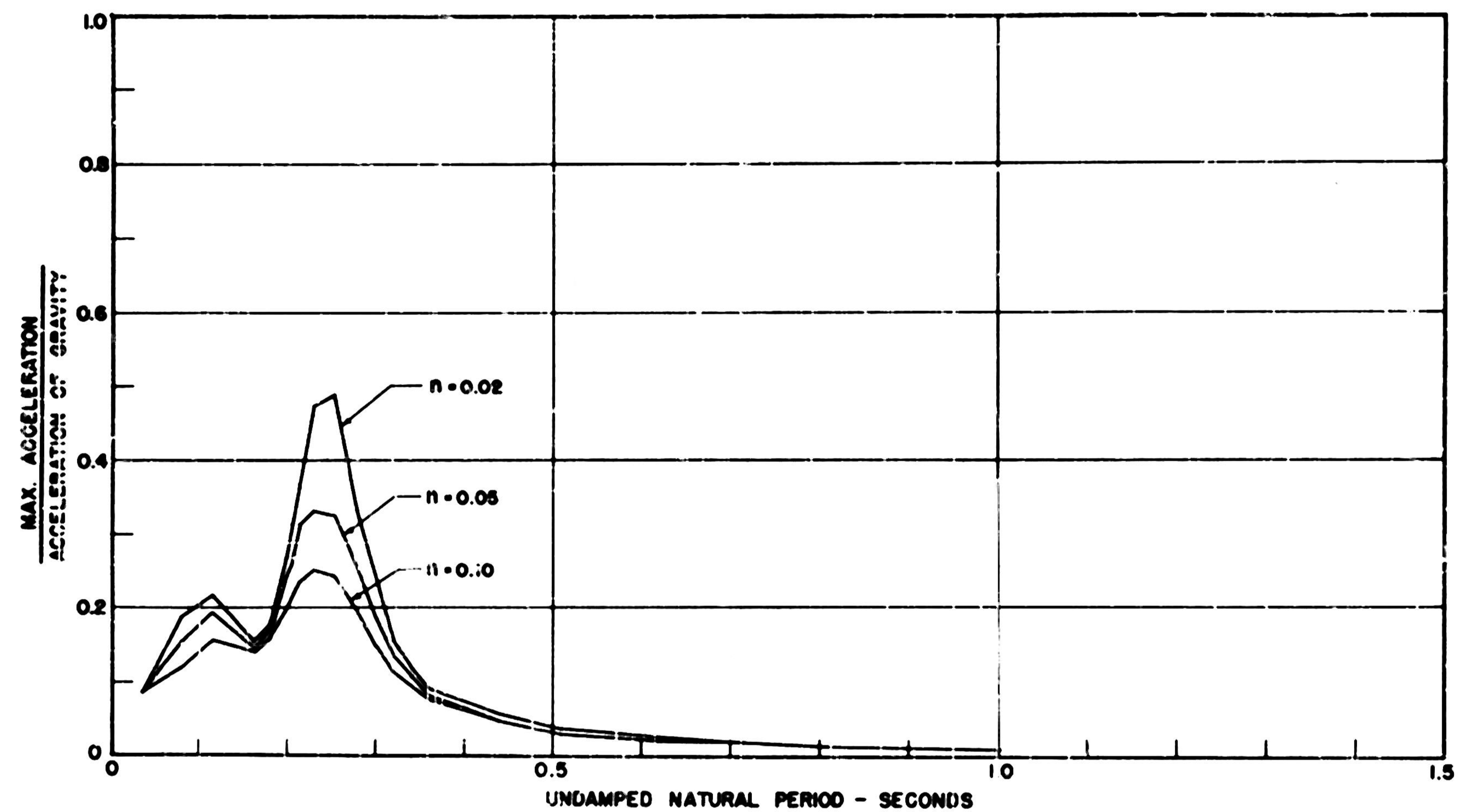

Figure 22. Acceleration spectra for Corona quarry blast, July 26, 1952. Component E-W.

It is instructive to contrast the blast spectrum to that of the El Centro earthquake (Figure 23), which may be considered as typical of strong-motion earthquakes. It can be seen, first, that the region of high response covers a much broader range of periods and, second, that the maximum response for the same degree of damping is nearly three times as great.

As more damping is added to the structure, the blast spectrum retains its characteristic peaks, wheras the addition of damping flattens the earthquake spectrum until the peaks virtually disappear. The two phenomena differ fundamentally in this respect. The ground shock from the blast can be expected to be damaging chiefly to structures within a restricted range of natural periods. The earthquake, on the other hand, can be expected to damage average structures of nearly all periods apt to be encountered in practice.

It is of some interest to examine the strength of the ground shock produced by the blast. If there are no geological peculiarities and if the shock propagates according to seismic observations*, it has been found that the acceleration will vary inversely with the square of the distance from the point of origin, that is:

$$a = \frac{C}{D^2} \qquad (1)$$

where "a" is the magnitude of an acceleration pulse, D is the distance from the point at which the ground acceleration is measured to the point of origin of the shock, and C is a constant. In the case of the blast the maximum horizontal acceleration was 0.13 g or 4.2 feet per sec^2, at a distance of approximately 400 yards or 1200 feet. For this acceleration pulse, equation (1) becomes:

$$a = 4.2 \left(\frac{1200}{D}\right)^2 \qquad (2)$$

where "a" is the maximum horizontal ground acceleration produced by the explosion, in feet per sec^2. The following table shows the corresponding maximum accelerations at varying distances:

---

*) B. Gutenberg and C. F. Richter, "Earthquake Magnitude, Intensity, Energy, and Acceleration", Bull. Seism. Soc. Amer., Vol. 32, p. 163, July 1942.

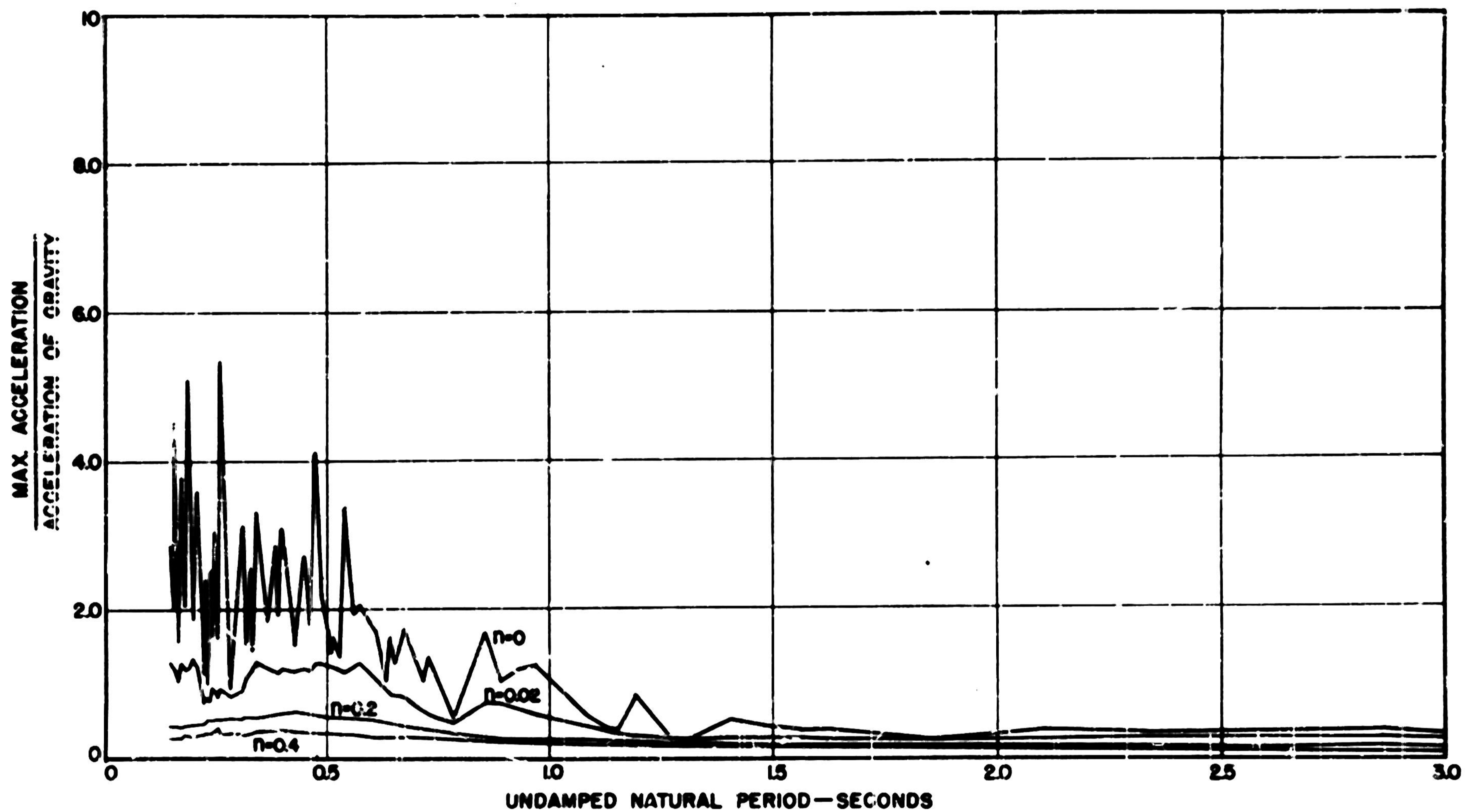

Figure 23. Acceleration spectra for El Centro, California; earthquake of May 18, 1940. Component N-S.

TABLE I

Maximum Horizontal Ground Acceleration

Distance from Origin in Feet	Acceleration in Feet per Sec^2
600	16.7
1,200	4.2
2,000	1.6
3,000	.67
4,000	.37
5,000	.24
10,000	.06

The foregoing expression may also be used in making an approximate extrapolation to larger quantities of explosive. In the case of 20,000 tons of explosive (nominal atomic bomb) as compared to the 185.7 tons used in the present test, if it is assumed that the amount of energy going into the ground per ton of explosive is the same and if the charge-weight scale as previously found for similar situations applies, there is obtained:

$$a = 4.2 \left(\frac{20,000}{185.7}\right)^{1/2} \left(\frac{1200}{D}\right)^2$$

or $$a = 44 \left(\frac{1200}{D}\right)^2 \qquad (3)$$

This indicates that the intensity of the ground motion for the 20,000 tons would be to that for the 185.7 tons as the ratio of 44 to 4.2 or 10.4 times as great. The corresponding maximum ground accelerations are shown in the following table:

TABLE II

Maximum Horizontal Ground Acceleration (20,000 Tons)

Distance from Origin in Feet	Acceleration in Feet per Sec^2
1,200	44.0
2,000	16.0
3,000	7.0
4,000	3.9
5,000	2.5
10,000	.63
20,000	.16

For the 20,000 tons explosion there would thus be an area of approximately 4000 feet radius shaken with a ground motion of intensity equal to or greater than a moderately strong earthquake.

VI. Measured Building Motion

A photograph of the record obtained on the concrete floor slab at elevation 45' - 8" above the first floor level is shown in Figure 24. A redrawn copy of the record is shown in Figure 25 to same scale as Figure 14. This record shows the east-west acceleration of the concrete floor slab. The maximum acceleration attained by the slab in this direction was 11.8% g. The recorded acceleration time picture agrees with what would be anticipated, namely, a building-up of the amplitude during the period of intense ground motion followed by a gradual diminution in amplitude of motion. At the instant of maximum acceleration the bracing in the building was stressed as if 11.8% of the weight of the floor slab were acting horizontally in an east-west direction.

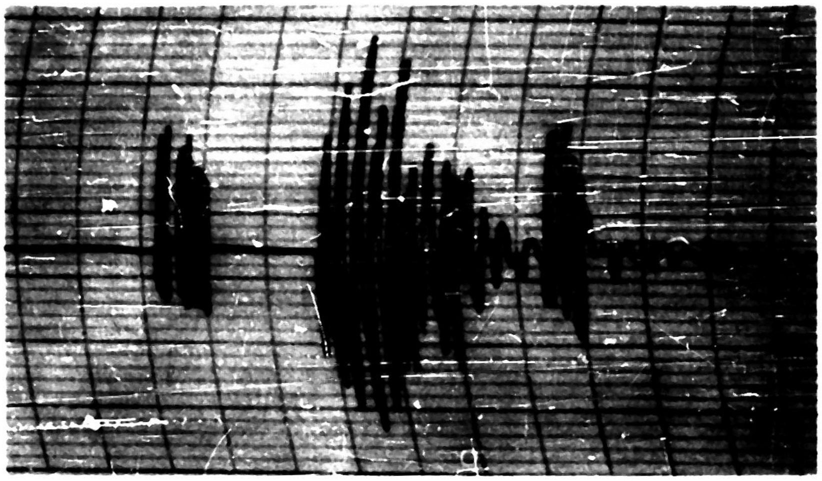

Figure 24. Photograph of east-west acceleration record from Brush recorder on upper floor.

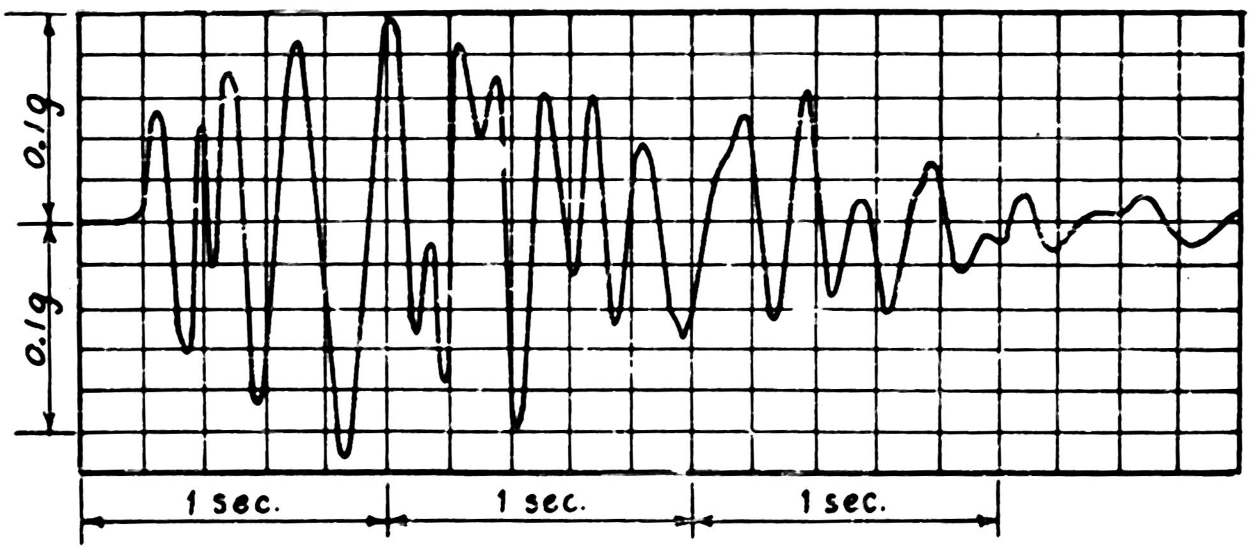

Figure 25. East-west acceleration at upper floor of the mill building.

VII. Computed Response of the Building

Having the record of the ground acceleration and knowing the properties of the structure, it is possible to compute the acceleration of the concrete slab. Such calculations have been made for the present test, using the Electric Analog Computer at the California Institute of Technology*.

The computations were carried through for a one degree of freedom system consisting of the concrete slab with its lateral bracing. The stiffness of the cross-bracing treated as vertical cantilever trusses, was computed to be 137,000 pounds per inch, that is, a horizontal force of 137,000 pounds applied in the east-west direction would displace the slab 1 inch laterally. This computation neglects the rod bracing which is in the adjacent panel as shown in Figure 4, so that the true stiffness may be somewhat larger. The weight of the slab, beams and tributary loads is approximately 180,000 pounds. The approximate natural frequency of vibration is computed to be:

$$f = \frac{1}{2\pi}\sqrt{\frac{137{,}000 \text{ lbs/in}}{180{,}000 \text{ lbs}} \times 386 \text{ in/sec}^2}$$

$$= 2.75 \text{ cycles per second}$$

The period of vibration is the reciprocal of the frequency, or 0.36 second. On the analog computer it was found that the calculated motion most nearly reproduced the measured motion of the slab when the natural frequency of vibration was adjusted to be 2.9 cycles per second. This agrees with the above calculation within the accuracy of the computation.

Figure 26 is a photograph of the cathode-ray tube of the Analog Computer; the upper trace shows the exciting ground motion, as reproduced in the form of an electric voltage for use in the computer. Comparison with Figure 14 will show that the essential

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*) See for instance, G.W. Housner and G.D. McCann, "The Analysis of Strong-Motion Earthquake Records with the Electric Analog Computer", Bull. Seism. Soc. Amer.. Vol. 39, p.47, January 1949.

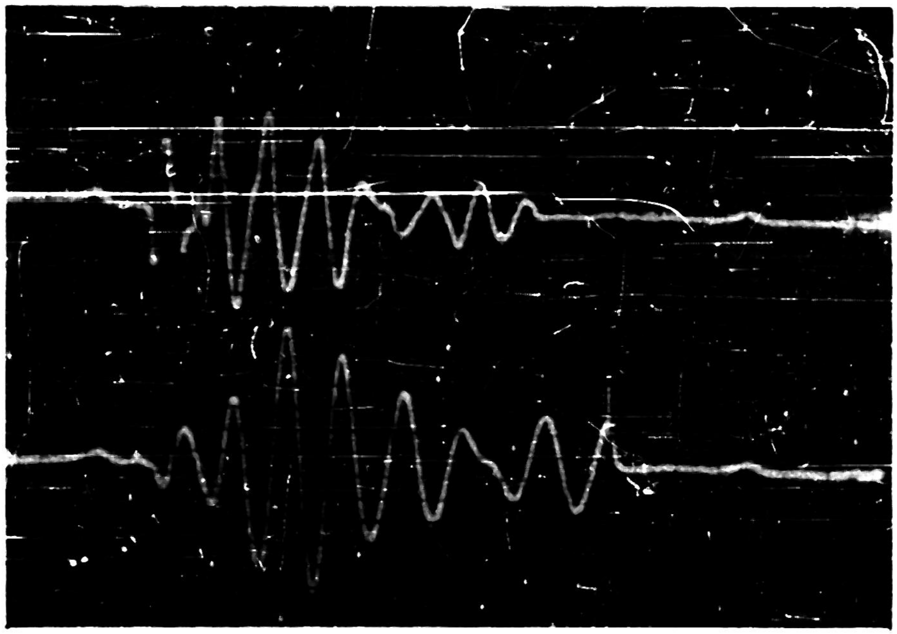

Figure 26. Photograph of computer solution. Upper trace: ground acceleration. Lower trace: east-west acceleration of the upper floor.

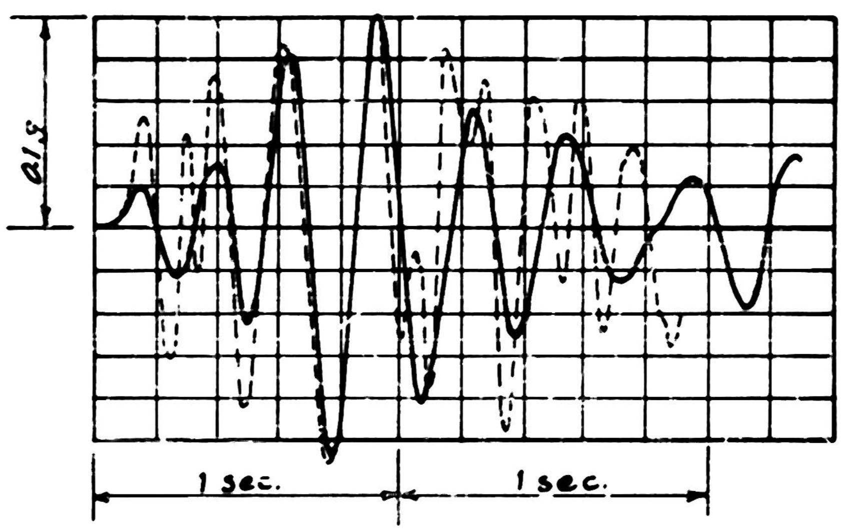

Figure 27. Comparison of computed and measured accelerations of the upper floor.

features of the ground acceleration have been faithfully reproduced. The lower trace of Figure 26 is the computed response of the floor slab, as simulated by a 2.9 cyc/sec, single degree of freedom oscillator. In Figure 27 this computed response has been redrawn to the scales used in Figures 14 and 25, and the measured response has been included in the same figure for comparison. It will be seen that the magnitudes and periods of the major peaks check very well, and that the general shapes of the curves are similar except for the high-frequency components in the measured record, which do not appear in the computed curve since only one degree of freedom was used for the dynamic model.

The computed response was not sufficiently sensitive to changes in damping of the system to permit a determination of the amount of damping in the building. This lack of sensitivity to damping is attributed to the short time duration of the ground shock. It has been found that damping is most effective in reducing the response of an oscillator to earthquake ground motion during the latter part of long-duration disturbances and least effective in disturbances of short duration*. The response shown in Figure 24 corresponds to a damping of about 1/2% of critical. Previous experience has indicated that low damping is to be expected in structures of this type, although the actual building damping is probably somewhat greater than 1/2%**. The maximum response of the structure can also be read from the spectra of Figure 22.

The response of the same structure was also computed for the ground motion of the El Centro, California earthquake of May 18, 1940 (Figure 19). A photograph of the response obtained with the Analog Computer is shown in Figure 28 for 8% of critical damping and in Figure 29 for 16% of critical damping. Since the El Centro earthquake was one of long duration and since the test building

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*) J. L. Alford, G. W. Housner and R. R. Martel, "Spectrum Analyses of Strong-Motion Earthquakes", First Tech. Report, ONR Project NR-081-095, Pasadena, California, August 1951.

**) J. L. Alford and G. W. Housner, "A Dynamic Test of a Four-Story Reinforced Concrete Building", Earthquake Engineering Research Institute, Pasadena, California, August 1951.

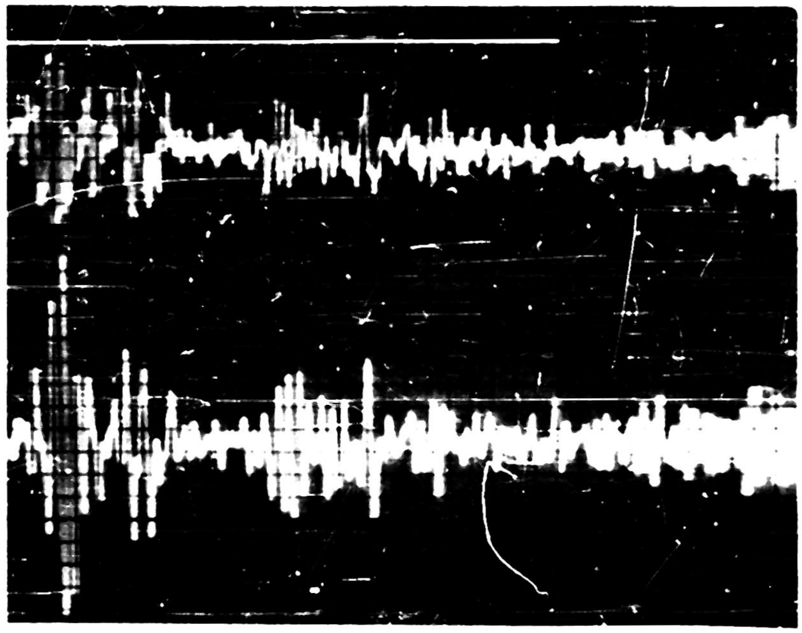

Figure 28. Upper trace: north-south ground acceleration at El Centro, California, May 18, 1940. Lower trace: computed acceleration at upper floor of mill building with 8% of critical damping.

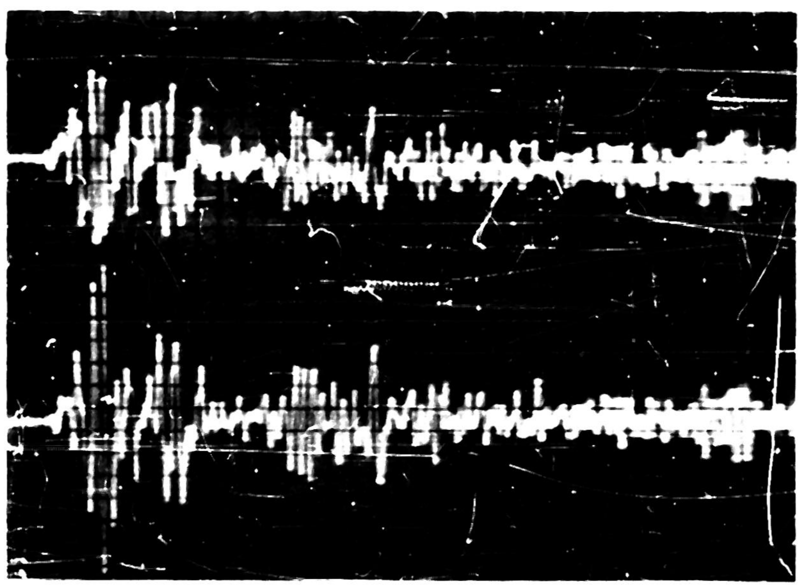

Figure 29. Upper trace: north-south ground acceleration at El Centro, California, May 18, 1940. Lower trace: computed acceleration at upper floor of mill building with 16% of critical damping.

damping is less than 8% of critical, the results are somewhat smaller than those which would be obtained using the true damping value.

The maximum acceleration of the slab (with 8% of critical damping) was 65% g, so that the maximum stresses produced in the cross-bracing would correspond to a lateral force equal to 65% of the weight of the slab. This assumes, of course, that the dynamic properties of the structure do not change during the motion. The maximum acceleration of the slab with 16% of critical damping was 45% g.

For the building under consideration lateral forces of the magnitude of 45-65% g would stress the bracing beyond the elastic limit so that the response of the test building to the El Centro earthquake would not have been as shown in Figures 28 and 29, for which the bracing must always remain within the elastic limit. It thus appears that if a structure of this type is to be designed to withstand strong earthquake motion without being stressed beyond the elastic limit, it must be designed for very sizeable lateral forces.

VIII. Relation to Building Design

The difficulty of computing the response of buildings to earthquakes is due to the complexity of most actual building structures rather than to any incompleteness or incorrectness in the dynamical theories applied. This is strikingly illustrated by the present tests, for in this case the test building was sufficiently simple so that computations could be made with confidence, and these computations were very closely checked by actual measurement. The theoretical calculations follow directly from basic physical principles and do not require the inclusion of empirical factors of any kind. Although it is true that a considerable amount of judgement must be used in representing a complex structure by a simplified model, it is of importance to know that a rational design method exists which can be directly applied to many simple structures and which will lead to realistic results.

The relatively large values of acceleration which were encountered in the present tests are of great significance in considering the effects of earthquakes on buildings. It has been supposed in some quarters that the actual lateral forces existing in structures during earthquakes must be somewhat less than indicated by analysis of records, in view of the relatively small amount of damage which has sometimes been noted.

The fact that in the present test the concrete slab of the building had a measured acceleration of 11.8% g produced by the blast and that the acceleration which would have been produced by the El Centro earthquake ground motion was computed to be 65% g raises an interesting question as regards the design of structures to resist earthquakes. That question is: What relation does the nominal design strength of a building bear to the actual ultimate strength of the building to resist earthquake forces? It is customary, in Southern California, to design structures to resist lateral forces of the order of magnitude of 10% g, but the ground shock from the blast produced lateral forces of this magnitude in the mill building and the strong ground motion of the El Centro earthquake produced computed forces much greater. Although there

has not as yet been a comprehensive test of structures designed to resist approximately 10% g, recent earthquakes have indicated that the 10% g design is, on the whole, not far out of line; at least there is no indication that all of the structures designed to resist such lateral forces are too weak. Because of the many uncertainties entering it is very difficult to compute backward from observed damage and determine reliably the magnitude of the lateral forces that would produce the damage. This is particularly true of most ordinary buildings. Occasionally, for certain special structures, it is possible to estimate with a fair degree of confidence the magnitude of the lateral force required to produce the observed damage. Sometimes this turns out to be in excess of 10% g. If we try to reconcile, on the whole, satisfactory performance of well designed structures during earthquakes with the accelerations to which they are subjected, we are led to certain tentative conclusions, as follows:

1. The ability of structures to resist earthquakes may be due to the fact that the elastic limit is exceeded and cracking and yielding dissipate the vibrational energy. That this does happen in some instances has been observed by measurements of the natural periods of vibration of buildings. It has been found in some cases that the period of vibration of a building was longer after the earthquake than it was before. This means that some of the lateral rigidity of the building was destroyed by the earthquake. In such cases the lateral strength of the building is reduced by the shock and presumably each succesive shock will further reduce it.

2. It may be that the actual ultimate strength of a building to resist lateral forces is much greater than the nominal design values and that the stresses during a strong earthquake are larger than the design stresses. It is well known that all structures designed to resist 10% g are not equally strong but that some of them are very much stronger than others. From this point of view, it may be that ordinary buildings have sources of strength which are not taken into account when the designs are made. The buildings may thus be much stronger in resisting lateral forces than is

ordinarily supposed and they may actually be subjected to forces greater than 10% g without being overstressed. In this case, if the satisfactory performance of the buildings depends upon sources of strength not recognized in the design, the factors of safety will be far from uniform. For example, the mill building which was tested has no source of strength to resist lateral forces other than the cross-bracing in the walls and this is taken into account in the design. On the other hand, a building with concrete or masonry walls, interior partitions, concrete fireproofing around beams, etc., may have various sources of strength which are not taken into account in the design and thus may be appreciably stronger than the design values indicate.

Undoubtedly both of the above factors enter into the problem and it is essential to the earthquake design problem that it be determined to what extent they do enter. It is important to know what the actual ultimate strength of a typical building is, and it is also important to know how this strength might be successively reduced by a series of shocks. Interesting information concerning this cumulative damage in repeated earthquakes may be available from a study of the damage in the Bakersfield area due to the series of earthquakes in July and August 1952.

IX. Summary

Accelerometer recordings were made of the three components of ground motion produced by the detonation of 370,000 pounds of explosive (Nitramon) at the Corona quarry of the Minnesota Mining and Manufacturing Company. The measurements were made in the sub-basement of a building, at a point approximately 400 yards from the point of detonation. The maximum accelerations were 8% g in the east-west direction, 10.5% g in the north-south direction, and 12.4% g in the vertical direction. The maximum resolved horizontal acceleration was 13% g. These acceleration magnitudes are comparable to those experienced in a moderately intense earthquake; however, the frequency spectrum of the blast shock is basically different from that of an earthquake.

The structure in which the measurements were made was a steel mill building having a heavy concrete floor slab at an elevation of 45' - 8" above the ground floor level. The east-west accelerations of this floor slab were also measured, and the maximum acceleration of the slab was found to be 11.8% g. By means of the electric analog computer, the response of the building to the recorded ground motion was computed and the response of the building was also computed for the recorded ground motion of a strong-motion earthquake. It was found that the earthquake ground motion would have produced relatively large accelerations and stresses in the building.

The large computed accelerations are discussed in the relation to acceleration values which are customary in design. It is concluded that the satisfactory performance of well-designed structures during earthquakes may have two explanations: first, that vibration energy is dissipated by stresses in excess of the elastic limit, with the result that hidden damage may occur; and second, that ordinary buildings may have sources of strength which are not taken into account in the design. The importance for earthquake-resistant design of answering these questions is indicated.

X. Acknowledgments

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